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# Integral Abutment Bridges: Current Practice in the United States And Canada

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# INTEGRAL ABUTMENT BRIDGES: CURRENT PRACTICE IN THE UNITED STATES AND CANADA

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## ABSTRACT

Integral abutment bridges have been gaining popularity among bridge owners as cost-effective alternatives to bridges with conventional joints. They reduce initial construction costs and long-term maintenance expenses, improve seismic resistance, and extend long-term serviceability. New York has been building them since the late 1970s, with a wide variety of details, and they have been performing well. For further improvement of New York's design practice, a comparative survey was undertaken across North America, focusing on design and construction of both substructures and superstructures. In all, 39 states and Canadian provinces responded, including 8 who said they had no experience with these bridges. Responses are analyzed and summarized in this report. Overall, integral abutment bridges are performing as well as, if not better than conventional bridges, but no uniform national standards exist for their design. Design practices and assumptions concerning limits of thermal movement, soil pressure, and pile design vary considerably among responding agencies. These decisions are based largely on past experience. Validity of these assumptions needs investigation by testing and analysis to ensure efficient and reliable design.

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# 1. INTRODUCTION

## A. BACKGROUND

Bridge structures expand and contract due to thermal strains, creep, and shrinkage. Traditionally, these movements have been accommodated by such components as expansion joints, roller supports, and expansion bearings. From past experience, bridge owners have found expansion joints and bearings to be expensive to install and requiring continuing maintenance. These joints are also prime sites for deterioration of the superstructure and substructure, due to deck leakage contaminated with de-icing chemicals. Based on an extensive literature review and survey, Wolde-Tinsae and Klinger (*1*) found in 1987 that expansion joints have negative economic impact in all phases of highway bridge service life, from design and construction through the inevitable subsequent maintenance burden. Funding and manpower are not readily available to solve these problems, which effectively shorten the service life of many structures.

A cost-effective alternative increasingly popular among bridge owners is the integral abutment bridge or integral bridge, terms generally referring to single-span or multiple-span continuous jointless structures with capped-pile stub-type abutments (*2*). These bridges offer several advantages over conventional structures and are currently used in more than 30 American states and Canadian provinces, and on the European continent (*2,3*). Benefits offered by integral bridges include reduced initial costs and long-term maintenance expenses, elimination of costly expansion joints and bearings, less deterioration due to de-icing chemicals, lower impact loads, improved riding quality, simple construction procedures, reduced substructure costs, and structural continuity to resist seismic events and overloads.

NYSDOT began building integral abutment bridges in the late 1970s and early 1980s, and by 1996 had completed 155 integral structures. For spans longer than 50 m (164 ft), use of steel H-piles with bending about the weak axis and a maximum allowable span length of 140 m (459 ft) have both been recommended (*4*). These bridges have principal features in common, although details have varied greatly, and a study by the NYSDOT Structures Design and Construction Division (*5*) concluded that with few exceptions, these structures have performed well. Current NYSDOT details for integral bridges are shown in Figures 1 and 2.

## B. THE PRESENT STUDY

In 1996, New York initiated a study to investigate the validity of certain design assumptions regarding soil pressure on abutments and load distribution among piles. This included surveying

**Figure 1. New York State detail for integral abutments with steel girders, including approach slab connection.**

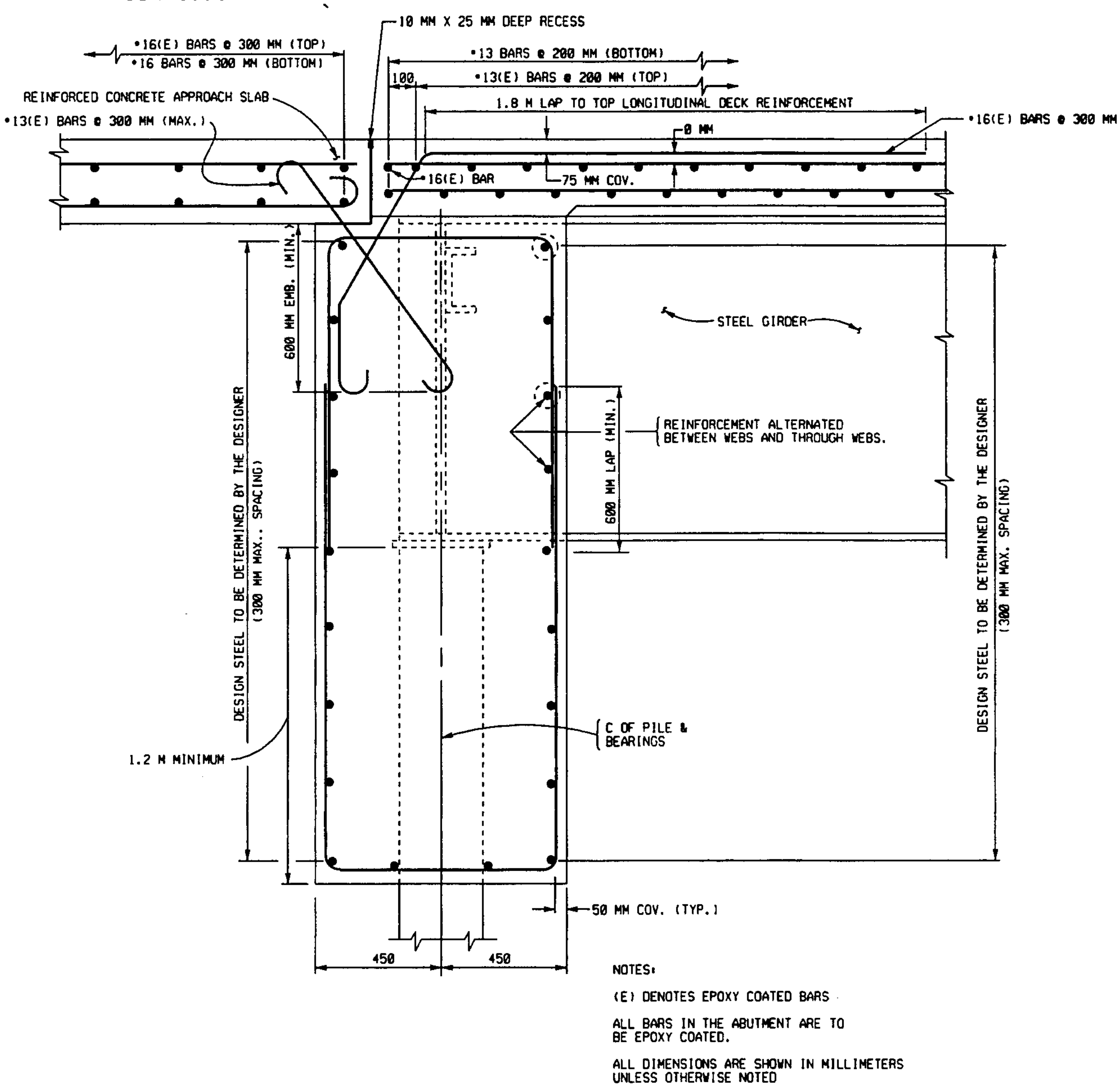
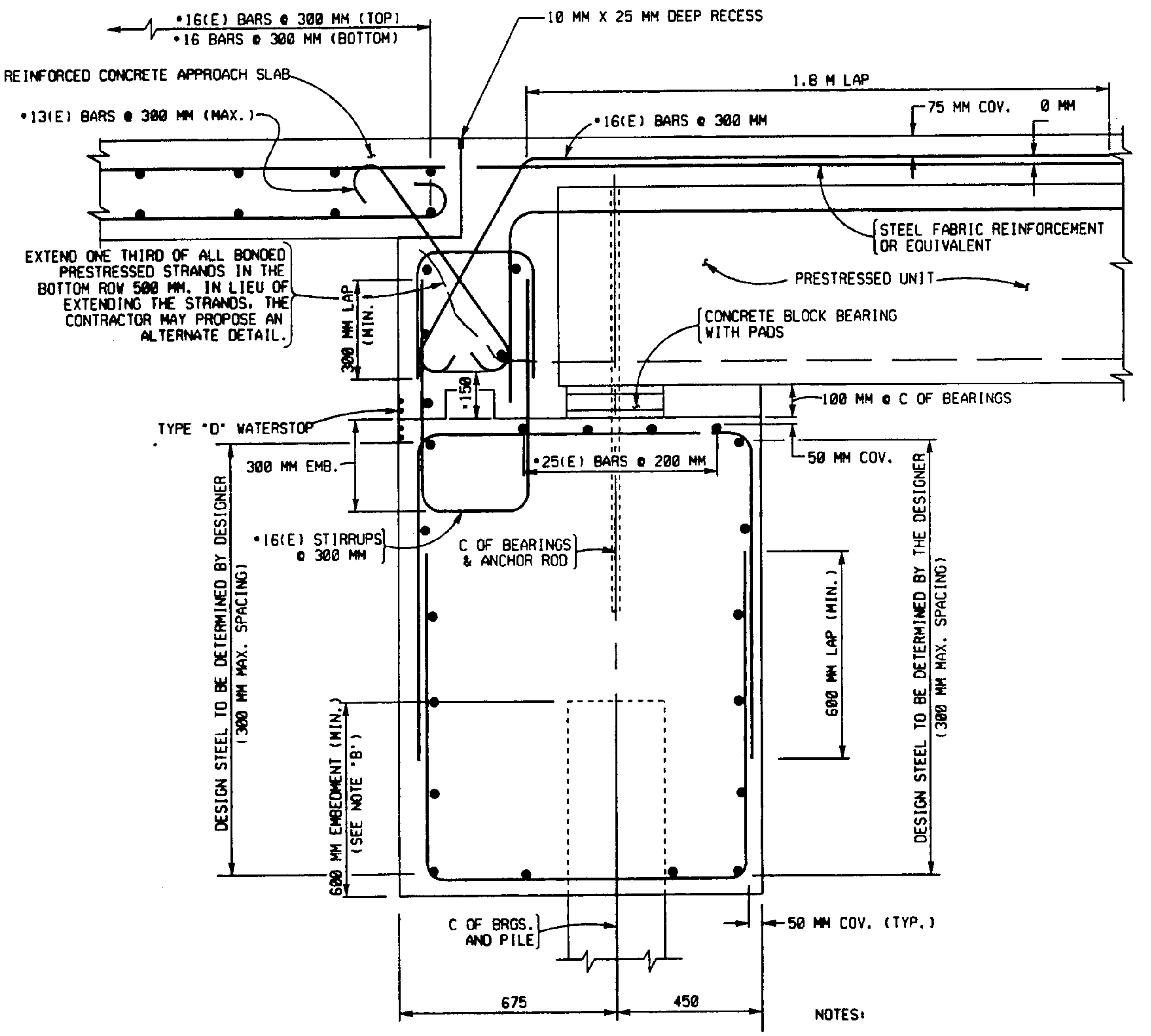


**Figure 2. New York State detail for integral abutments with prestresssed-concrete girders, including approach slab connection.**




NOTES:

(E) DENOTES EPOXY COATED BARS

ALL BARS IN THE ABUTMENT ARE TO BE EPOXY COATED.

ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE NOTED other transportation agencies nationwide and in Canada to document current practice in integral design and construction. A questionnaire was mailed, covering several aspects of integral bridge design, owner expectations, and in-service performance. The survey specifically requested details, which are summarized and compared here, concerning the following areas:

- Numbers and lengths of integral structures
- Earliest and most recent construction
- Analysis software used in design
- Thermal-expansion design limits and assumptions
- Soil-pressure assumptions
- Pile design
- Approach slabs
- Performance comparison of integral steel versus precast-concrete girders
- Special measures used to prevent scour or counteract earthquakes.

## 2. SURVEY RESULTS

The survey resulted in responses from 39 state or provincial transportation agencies in the United States and Canada, 8 saying only that they had no experience with integral abutment bridges. Overall, their general opinion of these bridges is very high, with most rating performance as "good" or "excellent." They appear to be performing well with minimal difficulties, but problems noted by agencies otherwise pleased with integral bridge performance included minor cracking, drainage at abutments, and settlement of approach slabs. Arizona was the only state with a negative opinion, having built 50 bridges, all requiring expensive repairs on the approaches -- thus, they do not recommend their use.

Table 1 summarizes responses regarding number built, earliest and most recent construction, length of longest structures, and computer software used in design. Construction was reported as early as 1905, and at the time of survey several more were being built. The longest precast-concrete-girder structure was a 358.4 m (1175 ft) bridge built in Tennessee. The longest steel-girder and cast-in-place concrete bridges were both built in Colorado, measuring 318.4 m (1044 ft) and 290.4 m (952 ft), respectively.

### A. THERMAL LIMITS

Most agencies use the following equation to estimate thermal movement:

$$\Delta \text{ Length} = \text{Length} \times \Delta \text{ Temperature} \times \text{Coefficient of Thermal Expansion} \qquad (1)$$

The extended length is usually apportioned equally to the two ends of the bridge. When using steel girders, most refer to AASHTO Specifications Article 10.2.2 and use 0.0000065 per deg F for thermal coefficient of linear expansion (*6*). A few use minor variations of this AASHTO recommendation. For concrete girders, agencies typically refer to AASHTO Article 8.5.3 which states that "the coefficient of thermal expansion and contraction for normal weight concrete may be taken as 0.000006 per degree Fahrenheit" (*6*). Minor variations of that value also were found.

Most agencies refer to AASHTO Article 3.16 to determine temperature ranges to be considered when evaluating an integral structure. In this Article, the range of temperature for metal structures is 0° to 120°F for moderate climates, and -30° to 120°F for cold climates. Concrete structures are assumed to rise 30°F and fall 40°F in moderate climates, and in cold climates these values change to a rise of 35°F and a fall of 45°F (*6*). Minor variations were found among respondents -- the Province of Quebec uses -40°C (- 40°F) and 15°C (59°F) for concrete structures.

Table 1. Summary of state and provincial responses.

| State or Province* | Total Built | First Built | Last Built** | Longest Built, m Steel Girder | Precast-Concrete Girder | Cast-In-Place Concrete Girder | Software Programs In-Use |
|---|---|---|---|---|---|---|---|
| AK | 50 | 1975 | 1995 | - | 41.2 | - | L-Pile, COM 624 |
| AZ | 50 | 1975 | 1985 | Use of integral abutments discontinued | | | |
| AR | 2 | 1996 | 1996 | 90.9 | - | - | In-house software |
| CA | Thousands | 1950 | 1995 | - | - | 122.0 | BDS |
| CO | 1643 | 1905 | 1996 | 318.4 | 339.2 | 290.4 | None |
| GA | 25 | 1975 | 1992 | 91.5 | - | 125.1 | P-Frame, Continuous Beam |
| IL | 350 | 1983 | 1996 | 61.0 | 91.5 | 36.6 | No special application |
| IA | Hundreds | 1962 | 1996 | 82.4 | 152.5 | 41.2 | None |
| KS | 1000 | 1935 | 1996 | 136.8 | 126.4 | 177.6 | None |
| KY | 260 | 1970 | 1996 | 89.1 | 122.0 | 31.7 | No special application |
| ME | 18 | 1983 | 1994 | 57.3 | 45.8 | 29.3 | No special application |
| MD | 18 | 1986 | 1996 | - | 15.9 | - | None |
| MA | 20 | 1930 | 1996 | 106.8 | 84.8 | 43.9 | GT STRUDL, L-Pile, FEA |
| MI | 6 | 1990 | 1996 | - | 147.9 | - | In-house software |
| MN | No records | 1958 | 1996 | 53.4 | 53.4 | 30.5 | Staad 3 |
| NV | Many | 1980 | 1996 | 77.8 | 33.6 | 84.2 | Standard girder design software |
| NH | 2 | 1993 | 1996 | 45.8 | 24.4 | - | - |
| NY | 155 | 1980 | 1996 | 93.3 | 68.3 | - | - |
| ND | 600 | 1960 | 1996 | 122.0 | 122.0 | 48.8 | None |
| NS | 2 | 1986 | 1996 | - | 38.0 | - | - |
| OK | 50 | 1980 | 1996 | - | 91.5 | - | No special application |
| OR | Unknown | 1940 | 1996 | - | 335.5 | - | STRUDL, BRIG2D |
| PA | 50 | 1946 | 1996 | 122.0 | 183.0 | - | Software being developed |
| QC | 1 | 1988 | 1988 | - | 78.1 | - | ANCAD, Structural Analysis 2D |
| SD | 818 | 1948 | 1996 | 112.9 | 209.2 | 106.8 | No special application |
| TN | 1000 | 1965 | 1996 | 175.4 | 358.4 | 189.1 | None |
| VT | 10 | 1975 | 1993 | 24.4 | - | - | In-house software |
| VA | 25 | 1982 | 1996 | 97.6 | 235.5 | - | None |
| WA | 1000 | 1965 | 1996 | - | 183.0 | 61.0 | In-House, SEISAB |
| WV | 60 | - | 1993 | 97.6 | 137.3 | 33.6 | None |
| WY | 1458 | 1957 | 1996 | 100.0 | 127.0 | 99.0 | BRASS, Risa, BDS |
| | | 1905 | 1996 | 318.4 max | 358.4 max | 290.4 max | |
| | Over 9773 Built | | | 24.4 min | 15.9 min | 29.3 min | |

* NS = Nova Scotia, QC = Quebec. Responses from Delaware, Louisiana, Manitoba, Mississippi, Montana, New Jersey, Newfoundland, and North Carolina indicated they had not built integral bridges.

** At the time of survey, several integral bridges were either under construction or in planning stages.

1 m = 0.305 ft

About 75 percent of responses indicate no design allowance for shrinkage. Of agencies that do allow for creep and shrinkage, most confine this to prestressed-concrete bridges or only the deck. Positive responses indicate 0.0002 as the coefficient of shrinkage for concrete, in accordance with AASHTO Article 8.5.4 (*6*).

Table 2 lists maximum allowable limits for thermal movement, bridge length, skew angle, tolerances for pile locations, abutment height, and stem height. A wide range of responses were received. Some specifications limit a structure's allowable thermal movement, and others limit bridge length. Both methods control a structure's thermal movement, but agencies controlling length usually allow larger movements. Others have no limits on length or thermal movement. Most agencies keep skew of integral bridges to less than 30°, but two set no skew limits. Maximum abutment height ranges from 0.9 m (3 ft) to no limit, and stem height from 0.3 m (1 ft) to no limit.

**Table 2. Maximum allowable limits.**

| | | Length, m | | | | | Height, m | |
|---|---|---|---|---|---|---|---|---|
| State or Province | Thermal Movement, cm | Steel-Girder | Precast-Concrete Girder | Cast-In-Place Concrete Girder | Skew Angle, deg | Tolerance for Pile Location, cm | Abutment | Stem |
| AK | – | – | 61.0 | – | 30 | 7.6 | – | – |
| AR | – | 91.5 | 91.5 | – | 15 | Per specs | No limit | No limit |
| CA | 1.3 | 31.1 | 50.9 | 50.9 | 21 | 10.2 | 4.3 | 2.7 |
| CO | 10.2 | 91.5 | 183.0 | 152.5 | No limit | 15.2 | No limit | No limit |
| GA | No limit | No limit | No limit | No limit | 30 | No specs | No limit | No limit |
| IL | No limit | 83.9 | 114.4 | 114.4 | 30 | Standard | No limit | No limit |
| IA | Limited by length | Undetermined | 152.5 | 152.5 | 30 | 7.6 | 0.9 to 1.5 | Length dependant |
| KS | 5.1 | 91.5 | 152.5 | 152.5 | 45 | 7.6 | By design | By design |
| KY | No limit | 91.5 | 122.0 | 122.0 | 30 | 15.2 | No limit | 0.9 m min pile cap |
| ME | 9.5 | 90.0 | 150.0 | 150.0 | 25 | 5.1 | 3.6 | – |
| MD | 2.5 | – | 18.3 | – | 30 | 15.2 | 3.1 to 4.6 | 3.1 |
| MA | Not defined | 99.1 | 99.1 | 99.1 | 30 | 7.6 | Minimize | Minimize |
| MI | No limit | No limit | No limit | No limit | 30 | 15.2 | – | – |
| MN | No limit | 61.0 | 61.0 | 61.0 | 20 | No specs | 1.0 | 1.0 |
| NV | 2.5 | 76.3 | 122.0 | 122.0 | 20 to 45 | – | Design | Design |
| NH | 3.8 | 45.8 | 24.4 | – | 10 | – | – | – |
| NY | Limited by length | 140.0 | 140.0 | 140.0 | 30 | 2.5 | – | 0.3 to 0.6 |
| ND | Limited by length | 122.0 | 122.0 | 48.8 | 30 | No specs | 3.7 | 1.5 to 1.8 |
| OK | – | 91.5 | 122.0 | – | No skew | 15.2 | 3.1 | 1.8 |
| OR | No limit | No limit | No limit | No limit | 45 | No specs | No limit | No limit |
| PA | 5.1 | 91.5 to 122 | 122.0 | Not used | 20 | – | – | – |
| QC* | No limit | – | 78.1 | – | 20°15' | 5.0 | 3.0 | 1.9 |
| SD | Limited by length | 106.8 | 213.5 | 213.5 | 30 | 15.2 | No limit | – |
| TN | 5.1 | 130.8 | 244.0 | 244.0 | No limit | No specs | – | No limit |
| VT | Limited by length | 24.4 | – | – | 15 | Standard | No limit | No limit |
| VA | 3.8 | 91.5/45.8** | 152.5/79.3** | – | 30 | 7.6 | No limit | No limit |
| WA | No limit | Not used | 106.8 | 61.0 | 30 | 15.2 | – | 3.7 |
| WV | 5.1 | Movement is limited, not length | | | 30 | 7.6 | No limit | No limit |
| WY | 5.0 | 100.0 | 130.0 | 100.0 | 45 | 2.0 | No limit | No limit |
| Max | No limit | No limit | No limit | No limit | No limit | Per Specs | No limit | No limit |
| Min | 1.3 | 24.4 | 18.3 | 48.8 | No skew | 2.0 | 0.9 | 0.3 |

* QC = Quebec
** Lesser value used with maximum skew.
2.54 cm = 1 in., 1 m = 0.305 ft

About half the designers report no distress related to thermal movement. Of those who do, most problems are limited to cracking and approach slab settlement. In Alaska, unexpected adhesion of frozen soil is suspected as the cause of hairline cracking in integral backwalls. Others report cracking and spalling in bearing areas, both longitudinal and transverse cracking on approach slabs, and transverse cracking on decks in areas of piers and counter-flexure points. Michigan reports minor spalling in the end diaphragm when a wingwall is separated from the abutment with an open joint. Kansas found distress in some steel beams and has since revised their specifications for stiffer bearing details.

Most reported no unexpected behavior with respect to thermal movement. Kansas has found shrinkage to be an issue for prestressed bridges, and advises engineers to account for this in future construction. Colorado reports that "although there could be bowing up of girders/superstructure, it appears all movement takes place in the abutments. Difference in stiffness of the superstructure versus the abutment does not appear to have an effect." Michigan and Oklahoma have found less movement than expected, a possible explanation being that the method used to calculate thermal movement is overly conservative and may not accurately reflect field performance.

## B. SOIL PRESSURE

Passive soil pressure is the design factor most commonly used. Six agencies use a combination of active and passive soil pressures -- in those instances, soil is typically active for contraction and passive for expansion. Two agencies do not consider soil pressure within certain abutment size limits, and three do not consider earth pressure at all in their design. Uniform, triangular, and Rankine load distributions are used in design for soil pressure. Massachusetts has started determining design earth pressures based on tests of an instrumented full-scale wall (7). California and Alaska assume an ultimate passive soil resistance of 53 MPa (7.7 kips/ft$^2$) to prevent failure from seismic loading. North Dakota uses a loading of 6.9 MPa (1 kip/ft$^2$) on integral abutment walls, and Michigan performs soil pressure analysis with an in-house program.

Two-thirds of those responding have no special procedure to reduce soil pressure against abutments. New Hampshire and West Virginia have used loose, uncompacted fill, but New Hampshire attributed settlement of sleeper slabs to this loose fill and now requires compaction. Illinois reports excellent results using an uncompacted, porous, granular embankment with an underdrain. Kentucky uses granular backfill both behind and in front of the abutment. Oregon uses a soil-reinforced fill with a gap at the structure wall. Wyoming has had satisfactory performance with a 50 to 100 mm (2- to 4-in.) gap between the abutment and a geotextile-reinforced backfill. Michigan used a high-density foam backing on one bridge. The performance of the foam is difficult to evaluate since the designer questioned whether the foam backing was necessary. Colorado typically uses a flow fill with a low-density foam and an expansion joint, providing for 50 to 100 mm ( 2- to 4-in.) movements at the end of the approach slab. Kansas installs approach slabs tied to the bridge, along with strip drains.

About two-thirds of respondents indicate that effects of skew are not considered with respect to soil pressure. Three indicate that they currently have no integral abutment bridges with skew. Kansas considers effects of skew only if lateral motion is a concern. In Washington State, bearings are put on the bottom sides of girders at abutments to transfer loads due to earth pressure. In Colorado and Quebec, soil pressure is assumed to be normal to the abutment. Colorado has used battered piles to accommodate transverse loading. Maine assumes that loads on skewed abutments induce transverse forces and translation to the piles. Oregon designers are concerned that large skew will result in a large torque, with soil thrust loads not opposing one another.

## C. WINGWALLS

The survey showed equal numbers of agencies using either active or passive soil pressures in designing wingwalls for integral abutment bridges. The most common soil-pressure distribution used is triangular, but uniform, Coulomb, and Rankine distributions also appear in the responses. Some agencies have a standard soil-pressure load distribution. Kentucky and Nevada use an equivalent fluid pressure with a 0.6 m (2-ft) surcharge. North Dakota uses 4.5 MPa (650 psf) as a standard loading for integral wingwalls.

Typically, wingwalls are poured either monolithically with the abutment or rigidly tied into the abutment with reinforcement. In some cases, an open joint is left between the abutment and wingwalls. Arkansas and Colorado use a cantilever method for wingwalls. About two-thirds of

**Table 3. Pile orientation.**

| Pile Position | Agencies Responding |
|---|---|
| Weak axis perpendicular to direction of traffic | 10 |
| Weak axis in direction of traffic | 6 |
| Weak axis parallel to skew direction | 2 |
| Weak axis perpendicular to skew direction | 2 |
| Combinations: | |
| Weak axis in direction of traffic and perpendicular to skew direction | 2 |
| Weak axis perpendicular to direction of traffic and weak axis parallel to skew direction | 2 |
| Weak axis in direction of traffic and perpendicular to direction of traffic | 1 |
| Total | 25 |

respondents use U-shaped wingwalls, but seven report that they do not use such shapes. Of these seven, two note that their wingwalls are usually parallel extensions of the abutment, with a maximum-length limitation.

## D. PILE DESIGN

Responses showed that steel H- or HP-piles are the types most frequently used, but cast-in-place, prestressed, pipe, and concrete-filled steel-shell piles have also been used. About 75 percent indicate that they distribute loads uniformly to the piles. Other methods used include moment distribution, placing piles directly under beams, assuming that piles directly under the beams carry a higher portion of load, and considering the abutment a continuous beam. Structural analysis is also used to distribute eccentric loads to the piles.

Slightly less than half the respondents design piles solely for axial loads. Of these, four agencies perform lateral analysis if the design does not meet certain requirements, including possibility of scour, pile length, and maximum structure length. The other half conduct both axial and lateral analyses. Concerns were noted regarding combined axial loading and bending. Several agencies also analyze pile stresses. Designers use several methods to analyze lateral load. Some consider the pile fixed at a certain depth, and either fixed, pinned, or free at the top, depending on the connection. L PILE is used by some. Washington State designs piles based on geotechnical analysis performed by in-house engineers. Maine uses the following formula to determine rotational demand:

$$\text{Rotational Demand} = M_x/S_x + M_y/S_y \leq 0.55F_y \tag{2}$$

Pile orientations are summarized in Table 3. Four agencies differ from those listed in the table. Washington State typically alternates orientation from pile to pile. North Dakota places the weak axis parallel to the abutment face. Colorado typically placed the weak axis parallel to the skew direction, but for larger movements the weak axis may be oriented in the direction of movement. Quebec has used only pipe piles.

When asked if oversized holes are predrilled before pile driving and later backfilled with granular material, 18 of 30 agencies said no. Four say that holes are predrilled if certain conditions are met,

**Table 4. Pile predrilling details.**

| State | Hole Size | Min Hole Depth | Density Limits and Type of Fill Material |
|---|---|---|---|
| CA | 15.25 cm oversize | 1.5 m | 95% |
| CO | Varies with expected shrinkage and shortening | - | Bentonite slurry |
| IA | 10.15 cm greater than diagonal pile dimension | 2.4 m | Natural Bentonite (no limits) |
| KA | Pile size (45.7 to 61.0 cm) | - | Sand (no limits) |
| MI | - | Depth of engineered embankment plus min of 4.6 m in dense soils | Loosely placed Class II material |
| ND | Not specified (Oversized holes used in fill sections only) | Through fill if in place for at least 30 days; through fill plus 4.6 m into original ground if less than 30 days | Not specified |
| PA | Varies with pile size | 3.0 to 6.1 m | Clean dry sand |
| SD | Varies with pile size (for 25.4 cm pile, 38.1 cm min hole diam) | 3.0 m or to original ground, whichever is greater | Coarse dry sand, no density specified |
| WV | 2.5 or 5 cm greater than pile diameter | 4.6 m | No backfill |

Conversions: 2.54 cm = 1 in., 0.305 m = 1 ft

including short fixed piles, difficult driving conditions, piles in fill sections, and bridge length of more than 30 m (98.4 ft). Several provided details for size of the pile hole, minimum hole depth, type of backfill, and required density limits. Table 4 summarizes these responses. None indicate using a compressible material on piles to reduce earth pressure. Colorado has used a bitumen coating to reduce down drag on piles, but has not tried to reduce earth pressure.

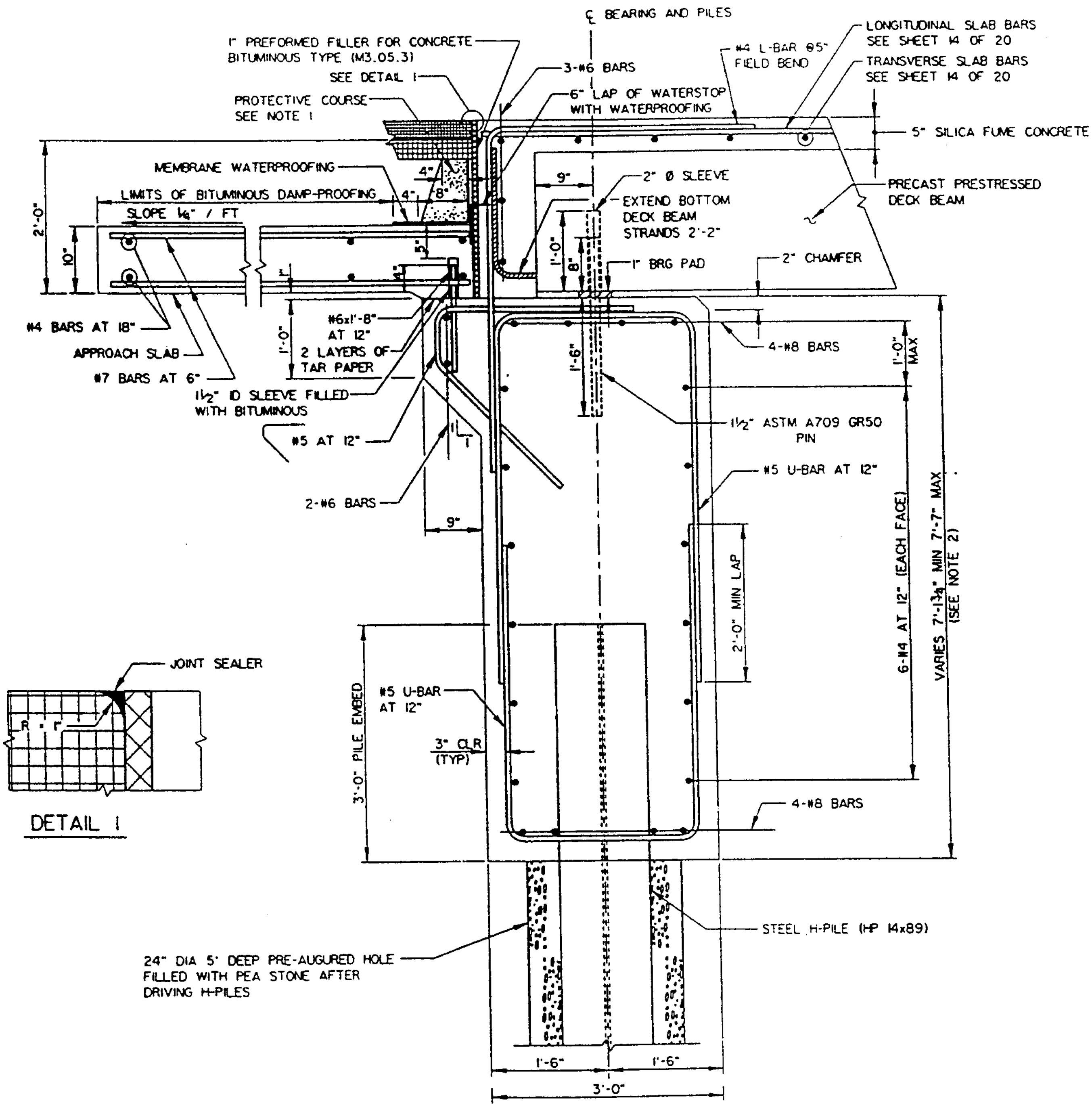
## E. APPROACH SLABS

Most agencies have similar approach-slab details, typically with the slab resting on a lip or corbel built into the abutment. Approach slab details differ in the connection of these slabs to the integral structure. One design uses reinforcement to connect the approach slab to the bridge deck. Most often, longitudinal reinforcement from the deck extends into the approach slab. Another detail has reinforcing steel connecting the corbel or lip of the abutment to the approach slab. Other designs do not rigidly connect the approach slab to the abutment, but allow the approach slab simply to rest on the corbel or lip. The two design methods are used with about equal frequency, with a reinforcing steel connection appearing slightly more often than a free-floating approach slab. Massachusetts has a unique detail where the concrete approach slab is below finished grade and not part of the wearing surface. The approach slab is backfilled and paved. This Massachusetts approach slab detail is shown in Figure 3.

Most agencies rate performance of their approach slabs as at least satisfactory. Typical approach slab flaws include settlement, transverse or longitudinal cracking, and cracks in asphalt overlays at the ends of approach slabs. When the slab is left to float on the corbel, abutment concrete has deteriorated due to runoff coming through the joint. North Dakota has used a system in which the approach slab rests on a lip built into the abutment, but their design has had failures and this detail has been modified. Washington State reports difficulties with approach slabs when bridge length is more than 106.8 m (350 ft).

Generally, three methods of expansion control were reported. In some cases, the expansion joint is placed at the far end of the approach slab, and in others between the abutment and approach slab. A third method uses no expansion joints. All three methods are considered to perform satisfactorily.

**Figure 3. Massachusetts approach slab detail for integral abutment bridges.**



A wide range of criteria are used in selecting type of expansion joint. Desirable characteristics are durability, movement capacity, easy maintenance, resistance to damage from snow-removal operations, and moderate cost. Several responses indicate that expansion joint types are specified depending on bridge length, while others analyze length and then write specifications based on anticipated movement.

## F. STEEL VS. CONCRETE GIRDERS

When asked whether performance of steel girders differs from that of prestressed-concrete girders, 17 agencies had enough experience with both types for comparisons. Thirteen find no difference, but four have observed some differences. Colorado notes that steel bridges experience more rotation at bearings, causing some cracking and spalling in bearing areas of the abutment diaphragm. Kansas reports that steel girders move more and that concrete girders shrink. New York has found less deck cracking on steel-girder integral bridges (*8*). Oregon states that "bridges with prestressed or post-tensioned concrete will experience creep shortening after construction."

Typical details of a steel-girder/integral-abutment/pile connection have the pile cast into the abutment. The steel girder rests on a bearing pad above the pile, and the bearing area is then cast into the abutment. Some details include anchor bolts connecting the lower flange of the steel beam to the abutment. Another method is rigidly connecting the steel girder by welding it to a pile. Often, holes or notches are put in the stringer web to provide space for continuous transverse steel reinforcement. North Dakota's detail includes a keyway in the concrete below the beam. Seven of 30 responses indicated having built no integral abutment bridges with steel beams. New York State's detail for integral bridges with steel girders is shown in Figure 1.

When prestressed-concrete girders are used, piles are also cast into the abutment and the concrete beams typically rest on bearing pads on the abutment stem. Bearing pads are usually elastomeric or grout, with the beam then cast integrally into the abutment. Additional details include using grouted pins to connect the beam to the lower portion of the abutment. Sometimes galvanized pipe runs through the web to provide space for transverse reinforcement. Only one agency responded that they had not built integral bridges with prestressed-concrete beams. The New York State detail for integral bridges with prestressed-concrete girders is shown in Figure 2.

Colorado and Tennessee have converted several conventional abutments into integral abutments. Illinois, Kansas, Oklahoma, South Dakota, and Wyoming also have rehabilitated abutments into integral form. North Dakota is doing this on a limited basis. In Massachusetts, new integral abutments have been constructed behind existing abutments which remain in place to retain fill.

## G. SCOUR AND EARTHQUAKES

Responses concerning scour indicate that integral abutment bridges are evaluated in the same manner as traditional bridges. Typically, all structures over water are evaluated for scour, and conventional methods for scour prevention are also used for integral abutment bridges, including analysis to ensure sufficient pile embedment and erosion prevention using rip-rap. Typically, no special measures are reported to counter seismic loadings on integral abutment bridges, which are designed to meet local codes. Massachusetts evaluates seismic loadings on integral abutment bridges with finite-element analysis. In Oregon, approach slabs have hooked dowels to restrain earthquake movement of the slab with respect to the bridge.

# 3. CONCLUSIONS

Integral abutment bridges are being built by at least 30 agencies. Bridge owners rate their performance as "good" or "excellent," and have experienced only minor problems. Issues appear concerning cracking and settlement of bridge approaches. Design practices and assumptions relating to thermal limits, soil pressures, and piles vary considerably among responding agencies. These decisions are based largely on past local experience and thus are empirical in nature. Validity of these design assumptions should be investigated by testing and analysis to ensure efficient, reliable design.

Agency limits on thermal movement vary from 13 mm (½ in.) to an undefined maximum. Skew angle constraints vary from 0° to no set limit, but most agencies limit skew of an integral structure to 30° or less. Major differences exists in methods of designing integral abutments to resist soil loading. Although passive soil pressures are most often used, soil load assumptions and distributions vary. HP piles are frequently used for integral structures, but other types have also been used, including pipes and cast-in-place piles. Variations in pile analysis concern load distribution. Uniform loading, moment distribution, and assuming the abutment to be a continuous beam are among the methods used to distribute loads to piles.

Approach slabs are typically designed by either of two methods. In one, the slab rests on a lip or corbel in the abutment. In the other, the approach slab is either connected to the integral structure with reinforcing steel, or left to float on the corbel. Flaws have been reported for both methods. When the slab is rigidly connected to the structure, pavement has cracked at the far end of the approach slab. When the slab is left to float on the corbel, abutment concrete has deteriorated due to runoff coming through the joint. Differences are generally minor between structures having steel or concrete girders. Larger movements have been noted when using steel, and shrinkage has occurred in concrete girders.

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## REFERENCES


1. Wolde-Tinsae, A.M., and Klinger, J.E. *Integral Bridge Design and Construction.* Report FHWA/MD-87/04, Maryland Department of Transportation, January 1987.

2. Burke, M.P., Jr. "Integral Bridge Design Is On the Rise." *Modern Steel Construction,* Vol. 30, No. 4 (July-August 1990), pp. 9-11.

3. Broms, B.B., and Ingelson, I. "Lateral Earth Pressure on a Bridge Abutment." In *Proceedings of the Fifth European Conference on Soil Mechanics and Foundation Engineering, Madrid, 1972*, Vol. 1, pp. 117-23.

4. "Bridge Detail (BD) Sheets IA1 thru IA3, Integral Abutments." Engineering Instruction 96-035, Structures Design and Construction Division, New York State Department of Transportation, December 1996.

5. *Integral Bridge Abutments: Report on Condition and Performance.* Structures Design and Construction Division, New York State Department of Transportation, December 1995.

6. *Standard Specifications for Highway Bridges.* Washington: American Association of State Highway and Transportation Officials, 1996 (16th ed.).

7. Personal communication from Daniel S. Crovo, P.E., of the MassHighway Bridge Section, Boston, MA.

8. Alampalli, S. and Yannotti, A.P. "*In-Service Performance of Integral Bridges and Jointless Decks.*" Transportation Research Record 1624, Transportation Research Board, pp.1-7, 1998.